# Performance of Earth Retention System, St. Louis Data Center

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SYNOPSIS The design and performance of the earth retention system for a 35 ft deep excavation in medium clay is described. The earth retention system consisted of soldier piles and lagging with tieback anchors. One level of tiebacks included helical anchors installed in loose to medium dense sand. Behavior of the helical anchors in contrast to conventional drilled-in anchors is discussed.

## INTRODUCTION

The St. Louis Data Center is located in downtown St. Louis, Missouri and occupies the entire city block bounded by Pine, Chestnut, 8th, and 9th Streets (Figure 1). The general excavation extended to el. 14± (depth of 32 to 37 ft below grade). The exposed height of the earth retention system on the south and southwest edges of the excavation was about 32 to 35 ft (Figure 2). Localized excavations for pile caps and elevator shafts extended up to 10 ft deeper. A berm at el. 26 was left in place along the north, east, and northwest sides of the excavation making the exposed height of the earth retention system about 25 ft in those areas.

## NEIGHBORING STRUCTURES

There are existing buildings across the street on three sides of the site (Figure 1). Two are supported on rock bearing deep foundations; others are supported on shallow foundations bearing from 10 to 20 ft below grade. A 100-year old masonry railroad tunnel supported on footing foundations bearing about 25 ft below grade parallels the east edge of the site below 8th Street. Next to the railroad tunnel, the general excavation did not extend below the invert elevation of the tunnel, although localized pile cap excavations near the tunnel extended 15 ft deeper. There are numerous utilities beneath the surrounding streets, including 19th century sewers and telegraph lines, as well as new fiberoptic cables.

## GENERALIZED SUBSURFACE CONDITIONS

General subsurface conditions are shown in Figure 3. There is rubble fill from demolished buildings from grade (el 46 to 51) to depths up to about 10 ft. The fill is underlain to a depth of about 25 to 35 ft below grade by firm to stiff low plastic silty clay (modified loess) having an average undrained shear strength ($S_u$) of about 1.2 ksf. Stiff to very stiff high plastic clay ($S_u$ of 2 to 3 ksf) extends from the base of the modified loess to a depth of about 35 ft below grade. There is an approximately 10 to 20 ft thick stratum of loose to medium dense fine silty sand beneath the high plastic clay.

The sand is underlain by interbedded glacio-fluvial deposits consisting of stiff high plastic clay, silt, and fine sand which extends to bedrock, 80 to 100 feet below grade. Bedrock consists of high quality limestone of the St. Louis Formation which supports pile foundations for the new building.

Open standpipe piezometers sealed in the modified loess and another group sealed in the sand stratum below the high plastic clay indicated two potentiometric surfaces. The pre-construction piezometric elevation in the modified loess ranged from el 37± at the north edge of the site to el 30± on the south. Piezometeric elevations in the sand stratum varied between el. 18 and 22.

## EARTH RETENTION SYSTEM

The earth retention system consisted of soldier piles and wood lagging with tieback anchors. Soldier piles were H sections (HP 12x63 to HP 14x117) driven to bedrock rather than being placed in drilled holes as is commonly done in St. Louis. Piles were driven to reduce loss of ground associated with drilling in cohesionless soils and to provide adequate axial capacity. Soldier pile spacing was typically 8± ft and 4-inch thick wood lagging "Contact Sheeting" spanned between soldier piles.

Special bracing was used inside the railroad tunnel below Eighth Street. Due to space limitations, the tunnel will not be discussed i this paper.

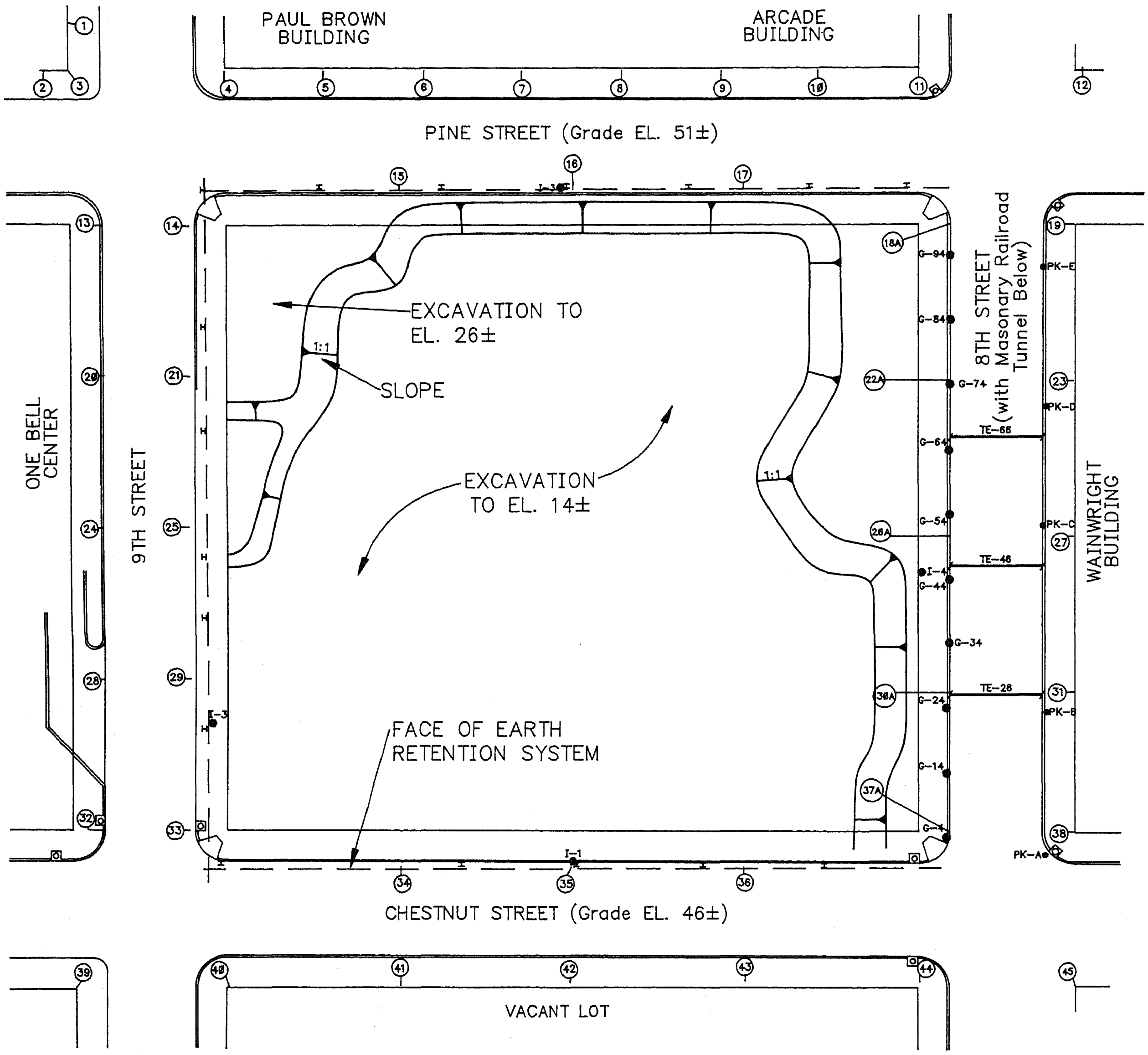


Fig. 1 – Site Plan Showing Earth Retention System and Instrumentation

LEGEND

- Survey Reference Point
- Soldier Pile Monitored by Survey
- Inclinometer
- TE-26 Tape Extensometer (Tunnel)

Ø 5Ø
Scale in Feet

N

Fig. 2 – Earth Retention System on 9th and Chestnut Streets

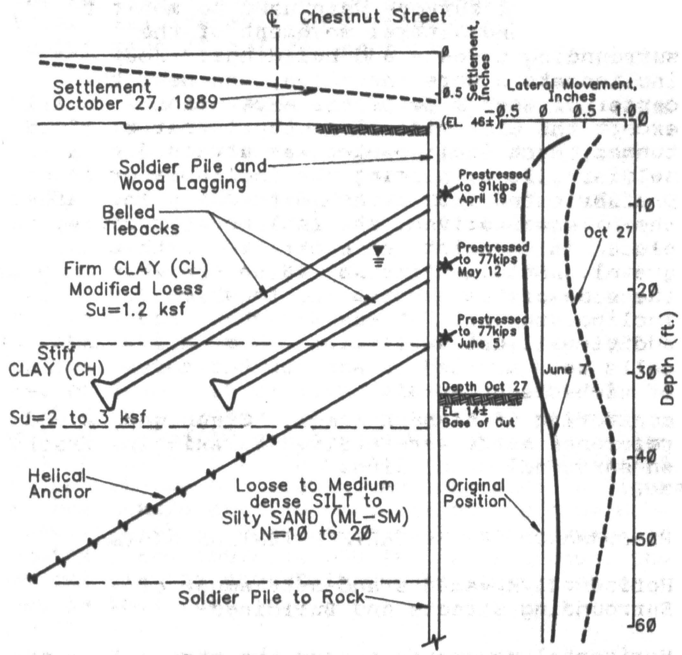



Fig. 3 – Cross Section at Inclinometer No. I-1 near the Center of Chestnut Street

## Tiebacks

To provide reasonable tieback spacings, and to permit use of standard high strength bars, tieback design loads were restricted to a maximum of 130 kips. This maximum tieback load required two levels of tiebacks on the north side and three levels on the south and west sides of the site. Tiebacks for the upper two levels of anchors (Levels A and B, respectively) were installed within the modified loess and very stiff high plastic clay. Drilled and under-reamed tiebacks were feasible for the A and B level anchors because the clay soils would stand open long enough to allow insertion of the anchor rod and placement of grout. Tieback shaft diameters varied from 18 to 24 inches and bell diameters ranged from 36 to 60 inches.

The third and lowest level anchors (Level C) required embedment in the loose to medium dense silty sand stratum. Drilled and under-reamed tiebacks were not feasible as the sand would be unstable during excavation and subsequent grouting. Use of pressure grouted anchors commonly used in sand was considered, but would have required different drilling equipment, and probably a second specialty contractor, than was used for the Level A and B anchors.

To permit use of the same drilling equipment for all anchors and to simplify construction, helical anchors were used for Level C. Each anchor consisted of a series of eight to ten 14 inch diameter helices at a spacing of about 3.5 ft along the anchor shaft (Figures 3 and 4). The anchors were put together in sections, each of which contained two or three helices. These anchor sections were bolted together to form the full 8 to 10 helix anchor. A standard threadbar extended from the helical portion of the anchor to the face of the excavation.

Performance tests were completed on several anchors, including helical anchors, and proof tests were completed on all anchors. After proof testing, anchors were locked off at approximately 70 percent of the design load.

Fig. 4 – Attaching Helical Anchor Section to Installation Rig

## Instrumentation

Geotechnical instrumentation and conventional rod and level surveys were used to monitor settlement and lateral movement of the surrounding streets and buildings. Four inclinometers were installed, one near the center of each side of the excavation. On all except the east side along the former railroad tunnel, each inclinometer was attached to a soldier pile by placing the inclinometer in a prefabricated slot attached to the pile. After the pile was driven, the inclinometer casing was placed in the slot and backfilled with pea gravel. Soldier Pile No. 46 on the west side of the side of the excavation, to which Inclinometer No. I-2 was attached, had additional instrumentation. Hydraulic load cells were mounted on each of the three levels of tiebacks on Soldier Pile No. 46 for long term monitoring of tieback loads. Crack gauges and reference marks were affixed to existing cracks on surrounding buildings.

## PERFORMANCE OF THE EARTH RETENTION SYSTEM

### Horizontal Movements and Settlement of Surrounding Streets and Buildings

Horizontal movements across the street from the excavation were less than 0.25 inch on Pine and Eighth Streets. On Chestnut Street the maximum movement of 0.48 inches occurred at the center of the block. On the 9th Street, movements were generally about 0.25 inch, although a value of 0.6 in occurred near the southwest corner of the block.

At the excavation face, horizontal movements were measured at the top of every sixth soldier pile. Lateral movements ranged from -0.6 inch (movement away from the excavation) on 9th street (Inclinometer I-2) to 0.84 inch near the center of both Pine and Chestnut streets. Typical values were about 0.6 inch.

Based on a percentage of excavation depth, the maximum horizontal movements across the street and adjacent to the excavation were 0.15 percent and 0.21 percent, respectively. These values are small compared to other sites documented by Clough (1975), Figure 5.

Settlement across the street from the excavation was typically less than 0.25 inch. The maximum of 0.36 inch occurred at the intersection of 8th and Pine streets.

Settlement measured 3 ft from the face of the excavation ranged from none to a maximum downward movement of 0.6 inch near the center of the Chestnut Street. Most values were around 0.5 inch. Interestingly, most settlement points showed about 0.25 inch heave during the winter months, presumably due to frost action.

As a percent of excavation depth, settlements across the street and adjacent to the excavation were about .09 percent to 0.15 percent, respectively. Again, these values are small compared to other sites (Figure 5).

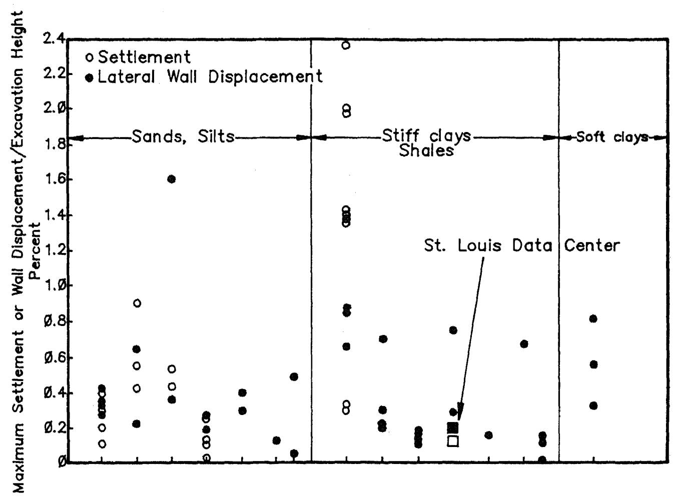


Fig. 5 – Some Movements of Tied-Back Walls as Reported by Clough (1975) Compared to the St. Louis Data Center

The comparatively small movements at this site are judged to be due primarily to the amount of prestress. Other contributing factors are probably the low plasticity and high strength of the retained soils and the presence of relatively high strength soils below the base of the excavation. The relationship between prestress and movement for this project compared to others noted by Clough (1975) is shown in Figure 6.

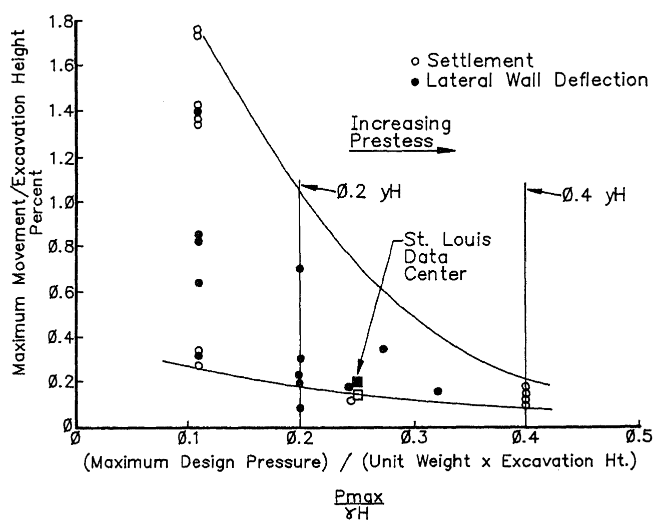


Fig. 6 – Effect of Prestress Pressure on Wall Movement for Clays. Pmax values based on Apparent Pressure Diagram by Terzaghi and Peck (1967) for Stiff Clays

**Lateral Movements (Inclinometers)**

Lateral movements for Inclinometers No. I-1 on Chestnut Street and No. I-2 on 9th Street (the deepest portions of the excavation) are shown in Figures 3 and 7, respectively. These data show clearly the effect of the anchors in restraining the top of the wall; in the case of I-2, the anchors actually pulled the wall away from the excavation. The largest horizontal movements occurred near the base and below the excavation, with the maximum value of about 0.9 inches on Chestnut Street.

Tieback Loads (Load Cells)

Load cell data from Soldier Pile No. 46 is plotted in Figure 7. Tieback loads were essentially constant (within 4 kips) after the excavation reached the full depth for the period of monitoring of about a month. Loads fluctuated daily about 2 to 3 kips due to changes in temperature. The apparent earth pressure measured by the load cells is compared with values recommended by Peck (1969) in Figure 7. The measured apparent earth pressure is near the middle of Peck's range and corresponds to a value of about 0.3$\gamma$ H where $\gamma$ is the saturated unit weight of the soil and H is the height of the retained soil. This apparent earth pressure is consistent with that reported by Jackson et al (1973).

HELICAL ANCHORS

Unlike drilled and under-reamed tiebacks which are routinely used in the St. Louis area little local data are available regarding use or performance of helical anchors.

Helical anchors were installed by turning them into place with a rig typically used for installation of drilled piers or drilled and under-reamed tiebacks. During installation, an electronic torque transducer mounted on the kelly bar of the drill rig monitored the torque applied to the helical anchor. Installation was terminated when the torque corresponding to the design ultimate pullout capacity was reached. The manufacturer's recommended correlation between ultimate anchor pullout capacity and torque is shown in Figure 8 along with performance test and proof test data. These test data indicate that the relation between pullout capacity and torque was quite variable and appeared to reach a plateau at approximately 14,000 ft-lbs of torque. As indicated in Figure 8, the manufacturer's correlation would be non-conservative in many cases due to the data scatter, and above 14,000 ft-lbs of torque, the correlation is uncertain due to limitation on the maximum test load.

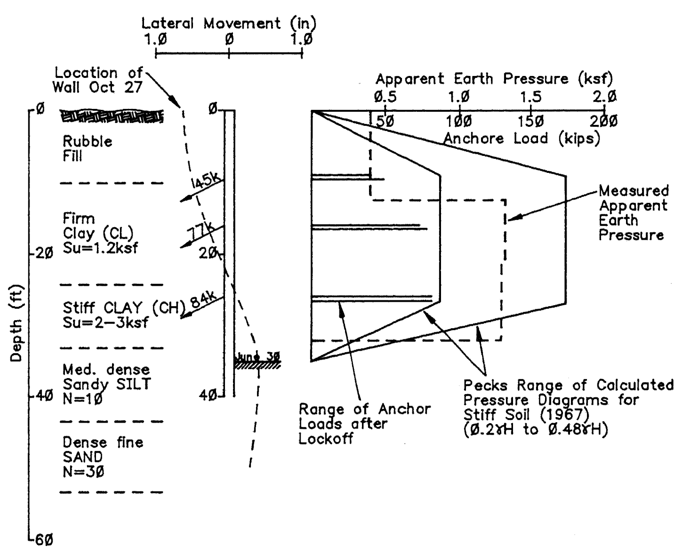


Fig. 7 – Measured Anchor Loads, Apparent Earth Pressures, and Lateral Movement at Soldier Pile 46

***Supplemental anchor installed June 15 bringing total lockoff load to 84k before excavating to full depth June 30.**

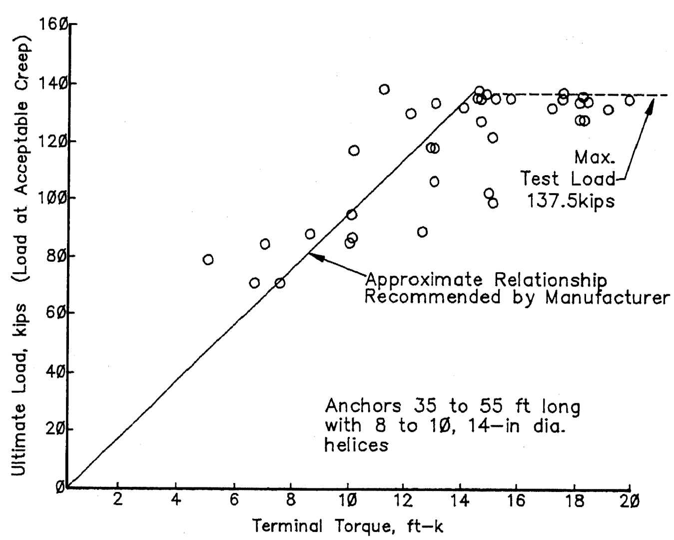


Fig. 8 – Relationship between Ultimate Anchor Capacity and Torque for Helical Anchors Installed in Medium Dense Sand

The results of a typical performance and creep test of a helical anchor embedded in loose to medium dense sand are shown in Figure 9. The test results indicate that about 0.5 inches of residual movement was needed to reach the design load of 110 kips. For a similar load some anchors required 1.5 to 2.0 inches of residual movement. This amount of movement is large compared to conventional tiebacks of similar length, and is attributed to removing slack from the bolted connections in the helical anchor. Another reason for the relatively large movement may be that helical anchors resist load primarily due to bearing on the surface of the helices rather than skin friction as do most conventional anchors. Movement required to develop bearing is generally larger than that to develop skin friction (Reese and O'Neill, 1988).

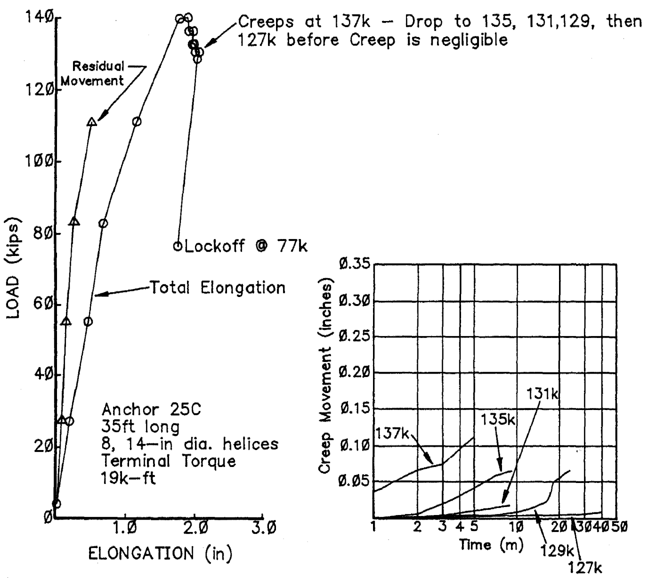


Fig. 9 – Performance and Creep Test on Typical Helical Anchor Installed in Medium Dense Silty Sand

The creep test results shown in Figure 9 are generally similar to test results on conventional anchors. The results indicate that the anchor capacity is about 127 kips compared to the design load of 110 kips. The results indicated that the helical anchors could not sustain the proof test load of 137.5 kips, but showed acceptable creep behavior at about 130 kips. This was typical of other helical anchors and suggested that the maximum practical design load for helical anchors used at this site was about 100 kips; slightly less than the 110 kips desired.

## CONCLUSIONS

1. Movements of the excavation bracing and neighboring streets and buildings were tolerable and small in relation to other published case histories. The relatively small movements are attributed to: 1) the amount of prestress applied to the earth retention system, 2) the low plasticity and high strength of the retained soil, and 3) the presence of relatively high strength soil below the base of the excavation.

2. Helical anchors in the sand stratum provided expedient anchorage that could be installed using conventional pier drilling equipment. The relationship between installation torque and ultimate pullout capacity is variable. Relatively large residual movements were needed to develop helical anchor capacity. The maximum practical design load was about 100 kips for the helical anchors used in the loose to medium dense sand at the site.

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## CONVERSION FACTORS

| To Obtain | Multiply | by |
|---|---|---|
| meters | feet | 0.3048 |
| milimeters | inches | 25.4 |
| kilopascals | ksf | 47.88 |
| meganewtons | kips | $4.448 \times 10^{-3}$ |